C5.1

5.1 SCOPE

5.2 DEFINITIONS

C5.2

The following shall be added:

Compression-controlled section – A cross section in which the net tensile strain in the extreme tension steel at nominal resistance is less than or equal to the compression-controlled strain limit.

Compression-controlled strain limit – The net tensile strain in the extreme tension steel at nominal strength at balanced strain conditions. See Article 5.7.2.1.

Extreme tension steel – The reinforcement (prestressed or non-prestressed) that is farthest from the extreme compression fiber.

Net tensile strain – The tensile strain in the extreme tension steel at nominal strength exclusive of strains due to effective prestress, creep, shrinkage and temperature.

Tension-controlled section – A cross section in which the net tensile strain in the extreme tension steel at nominal strength is greater than or equal to 0.005.

5.3 NOTATION

The following shall be added:

d_t = distance from extreme compression fiber to centroid of extreme tension steel (in.) (C5.5.4.2.1, C5.7.2.1)

ε_t = net tensile strain in extreme tension steel at nominal resistance (C5.5.4.2.1, C5.7.2.1)

C5.3

5.4 MATERIAL PROPERTIES	C5.4
5.4.1 General	C5.4.1
5.4.2 Normal and Structural Low-Density Concrete	C5.4.2
5.4.2.1 COMPRESSIVE STRENGTH	C5.4.2.1
The following shall be added to paragraph 3. The specified compressive strength for reinforced concrete shall not be less than 3.60 ksi (25Mpa).	
5.4.2.2 COEFFICIENT OF THERMAL EXPANSION	C5.4.2.2
5.4.2.3 SHRINKAGE AND CREEP	C5.4.2.3
5.4.2.3.1 General	C5.4.2.3.1
5.4.2.3.2 Creep	C5.4.2.3.2
5.4.2.3.3 Shrinkage	C5.4.2.3.3
5.4.2.4 MODULUS OF ELASTICIY	C5.4.2.4
5.4.2.5 POISSON'S RATIO	C5.4.2.5
5.4.2.6 MODULUS OF RUPTURE	C5.4.2.6
5.4.2.7 TENSILE STRENGTH	C5.4.2.7
5.4.3 Reinforcing Steel	C5.4.3
5.4.3.1 GENERAL	C5.4.3.1
5.4.3.2 MODULUS OF ELASTICITY	C5.4.3.2
5.4.3.3 SPECIAL APPLICATIONS	C5.4.3.3
5.4.4 Prestressing Steel	C5.4.4
5.4.4.1 GENERAL	C5.4.4.1
5.4.4.2 MODULUS OF ELASTICITY	C5.4.4.2
5.4.5 Posttensioning Anchorages and Couplers	C5.4.5
5.4.6 Ducts	C5.4.6
5.4.6.1 GENERAL	C5.4.6.1
5.4.6.2 SIZE OF DUCTS	C5.4.6.2
5.4.6.3 DUCTS AT DEVIATION SADDLES	C5.4.6.3
5.5 LIMIT STATES	C5.5
5.5.1 General	C5.5.1
5.5.2 Service Limit State	C5.5.2
5.5.3 Fatigue Limit State	C5.5.3
5.5.3.1 GENERAL	C5.5.3.1
5.5.3.2 REINFORCING BARS	C5.5.3.2
5.5.3.3 PRESTRESSING TENDONS	C5.5.3.3
5.5.3.4 WELDED OR MECHANICAL SPLICES OF REINFORCEMENT	C5.5.3.4

5.5.4 Strength Limit State
5.5.4.1 GENERAL

5.5.4.2 RESISTANCE FACTORS
5.5.4.2.1 Conventional Construction
Modify as follows:

- For ~~flexure and tension of~~ tension-controlled reinforced concrete sections as defined in Article 5.7.2.1…………..0.90
- For tension-controlled cast-in-place prestressed concrete sections as defined in Article 5.7.2.1……………………..…..0.95
- For ~~flexure and tension of~~ tension-controlled precast prestressed concrete sections as defined in Article 5.7.2.1……………………………..1.00
- For ~~axial~~ compression-controlled sections with spirals or ties, as defined in Article 5.7.2.1, except as specified in Article 5.10.11.4.1b for Seismic Zones 3 and 4 at the extreme event limit state………………………………..0.75

The balance of the bulleted items remain unchanged.

~~For compression members with flexure, the value of ϕ may be increased linearly to the value for flexure as the factored axial load resistance, ϕPn, decreases from 0.10 f′c Ag to 0.~~

For sections in which the net tensile strain in the extreme tension steel at nominal resistance is between the limits for compression-controlled and tension-controlled sections, ϕ may be linearly increased from 0.75 to that for tension-controlled sections as the net tensile strain in the extreme tension steel increases from the compression-controlled strain limit to 0.005.

This variation in ϕ may be computed for prestressed members:

$$0.75 \le \phi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \le 1.0 \quad (5.5.4.2.1\text{-}1)$$

and for nonprestressed members:

$$0.75 \le \phi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right) \le 0.9 \quad (5.5.4.2.1\text{-}2)$$

where:
c = distance from the extreme compression fiber to the neutral axis (in.)
d_t = distance from the extreme compression fiber to the centroid of the extreme tension steel element (in.)

For tension-controlled partiall prestressed components in flexure with or without tension….

The balance of the existing text remains unchanged

C5.5.4
C5.5.4.1

C5.5.4.2
C5.5.4.2.1
Add the following to Article C5.5.4.2.1 prior to the existing text:

In applying the resistance factors for tension-controlled and compression-controlled sections, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

In editions of and interims to the LRFD specifications prior to 2005, the provisions specified the magnitude of the resistance factor for cases of axial load or flexure, or both, it terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross section, at nominal strength. The background and basis for these provisions are given in Mast (1992) and ACI 318-02.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

For sections subjected to axial load with flexure, factored resistances are determined by multiplying both ***Pn*** and ***Mn*** by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in Article 5.7.2.1 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Figure 5.5.4.2.1-1. The concept of net tensile strain ε_t is discussed in Article C5.7.2.1. Classifying sections as tension-controlled, transition or compression-controlled, and linearly varying the resistance factor in the transition zone between reasonable values for the two extremes, provides a rational approach for determining ϕ and limiting the capacity of over-reinforced sections.

The balance of the existing text remains unchanged.

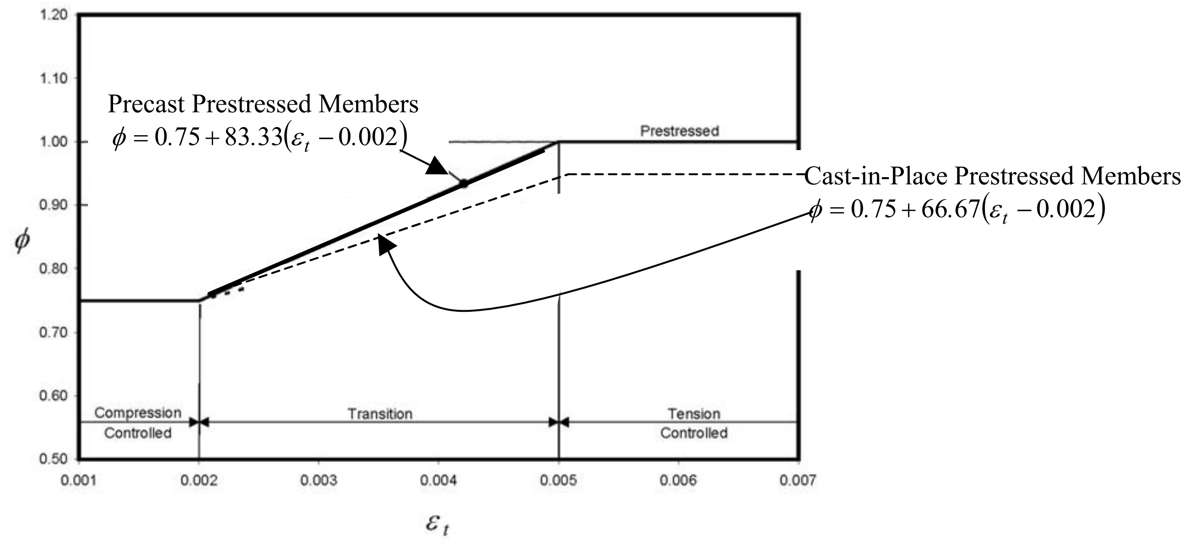

Figure 5.5.4.2.1-1 – Variation of ϕ with net tensile strain ε_t for prestressed members.

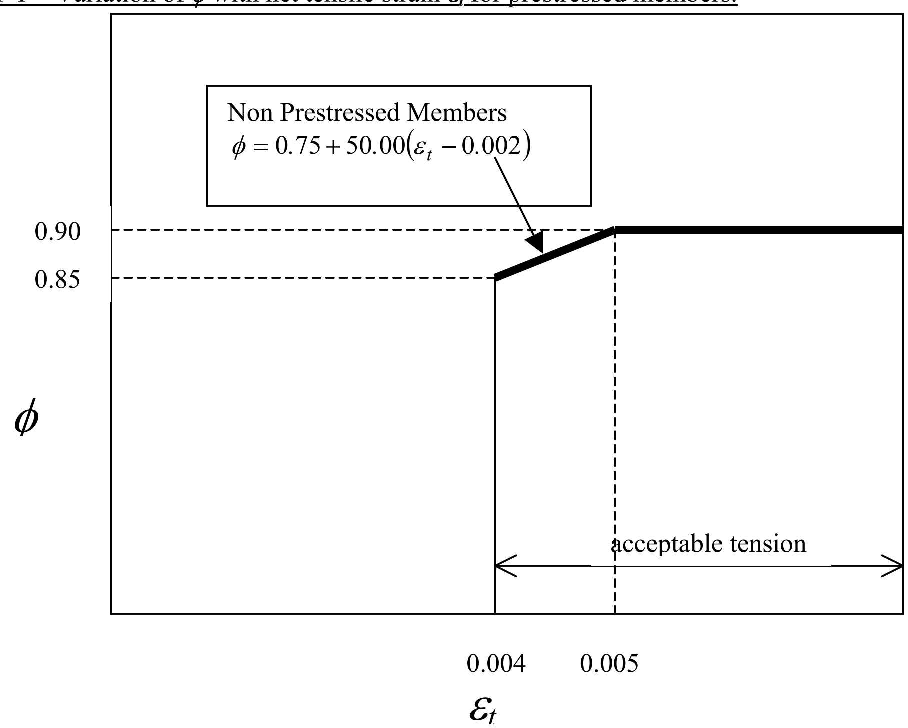

Figure 5.5.4.2.1-1 – Variation of ϕ with net tensile strain ε_t for Grade 60 reinforcement conventionally reinforced concrete members.

5.5.4.2.2 Segmental Construction

C5.5.4.2.2

5.5.4.2.3 Special Requirements For Seismic Zones 3 and 4

~~A reduced resistance factor for columns in Seismic Zones 3 and 4 shall be taken as specified in Article 5.10.11.4.1b.~~

C5.5.4.2.3

5.5.4.3 STABILITY

C5.5.4.3

5.5.5. Extreme Event Limit State

C5.5.5

Modify as follows:

The structure as a whole and its components shall be proportioned to resist collapse due to extreme events, specified in Table 3.4.1-1, as may be appropriate to its site and use. Resistance factors shall be 1.0.

C5.6

5.6 DESIGN CONSIDERATIONS

5.6.1 General

C5.6.1

5.6.2 Effects of Imposed Deformation

C5.6.2

5.6.3 Strut-and-Tie Model

C5.6.3

5.6.3.1 GENERAL

C5.6.3.1

5.6.3.2 STRUCTURAL MODELING

C5.6.3.2

5.6.3.3 PROPORTIONING OF COMPRESSIVE STRUTS

C5.6.3.3

5.6.3.3.1 Strength of Unreinforced Strut

C5.6.3.3.1

5.6.3.3.2 Effective Cross-Sectional Area of Strut

C5.6.3.3.2

5.6.3.3.3 Limiting Compressive Stress in Strut

C5.6.3.3.3

5.6.3.3.4 Reinforced Strut

C5.6.3.3.4

5.6.3.4 PROPORTIONING OF TENSION TIES

C5.6.3.4

5.6.3.4.1 Strength of Tie

C5.6.3.4.1

5.6.3.4.2 Anchorage of Tie

C5.6.3.4.2

C5.6.3.6

5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

C5.7

5.7.1 Assumptions for Service and Fatigue Limit States

C5.7.1

5.7.2 Assumptions for Strength and Extreme Event Limit States

C5.7.2

5.7.2.1 GENERAL

Add bulleted items to the end of the list in Article 5.7.2.1 as follows:

- Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.
- Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002.
- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections. For non-standard concrete flexural sections including girders, bent, caps, and deck slabs, the net-tensile strain in the extreme tension steel shall not be less than 0.004.
- The use of compression reinforcement in conjunction with additional tension reinforcement is permitted to increase the strength of flexural members.

C5.7.2.1

Add commentary to Article C5.7.2.1 to accompany the CA bulleted items as follows:

The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit of 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, as shown in Figure C5.7.2.1-1, using similar triangles.

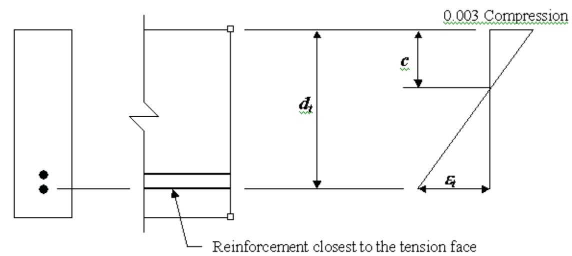

Figure C5.7.2.1-1 – Strain distribution and net tensile strain

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum reinforcement limit that was given as $c/d_e \leq 0.42$, which corresponded to a net tensile strain at the centroid of the tension reinforcement of 0.00414. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to all types of steel (prestressed and nonprestressed) permitted by this specification.

Unless unusual amounts of ductility are required, the 0.005 limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Article 5.7.3.5 permits redistribution of negative moments. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least 0.0075.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

5.7.2.2 RECTANGLULAR STRESS DISTRIBTION

C5.7.2.2

5.7.3 Flexural Members

5.7.3.1 STRESS IN PRESTRESSING STEEL AT NOMINLA FLEXURAL RESISTANCE

5.7.3.1.1 Components with Bonded Tendons
Modify Eqn 3 as shown:

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A'_s f'_y - 0.85 \cancel{\beta_1} f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}}$$

C5.7.3

C5.7.3.1

C5.7.3.1.1
Remove all of Article C5.7.3.3.1 and replace with:

In editions of and interims to the LRFD Specifications prior to 2005, Article 5.7.3.3.1 limited the tension reinforcement quantity to a maximum amount such that the ratio c/d_e did not exceed 0.42. Sections with $c/d_e > 0.42$ were considered over-reinforced. Over-reinforced nonprestressed members were not allowed, whereas prestressed and partially prestressed members with PPR greater than 50 percent were if "it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved." No guidance was given for what "sufficient ductility" should be, and it was not clear what value of ϕ should be used for such over-reinforced members.

The current provisions of LRFD eliminate this limit and unify the design of prestressed and nonprestressed tension- and compression-controlled members. The background and basis for these provisions are given in Mast (1992). Below a net tensile strain in the extreme tension steel of 0.005, as the tension reinforcement quantity increases, the factored resistance of prestressed and nonprestressed sections is reduced in accordance with Article 5.5.4.2.1. This reduction compensates for decreasing ductility with increasing overstrength. Only the addition of compression reinforcement in conjunction with additional tension reinforcement can result in an increase in factored flexural resistance of the section.

5.7.3.1.2 Components with Unbonded Tendons

Modify Eqn. 3 as shown:

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f'_y - 0.85 \cancel{\beta_1} f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w}$$

C5.7.3.1.2

5.7.3.2 FLEXURAL RESISTANCE

C5.7.3.2

5.7.3.2.1 Factored Flexural Resistance

For flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used ~~and the tendons are bonded~~ and where the compression flange depth is less than ~~c~~ $a=\beta_1 c$, as determined in accordance with Equations 5.7.3.1.1-3, 5.7.3.1.1-4, 5.7.3.1.2-3 or 5.7.3.1.2-4, the nominal flexural resistance may be taken as

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A'_s f'_y\left(d'_s - \frac{a}{2}\right) + 0.85 f'_c (b - b_w) \cancel{\beta_1} h_f \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

where:

~~β_1 = stress block factor specified in Article 5.7.2.2~~

C5.7.3.2.1

:

5.7.3.2.2 Flanged Sections

C5.7.3.2.2

Delete current Article C5.7.3.2.2 in its entirety, including Figure C5.7.3.2.2-1, and replace with:

In previous editions and interims of the LRFD Specifications, the factor β_1 was applied to the flange overhang term of Equations 1, 5.7.3.1.1-3 and 5.7.3.1.2-3. This was not consistent with the original derivation of the equivalent rectangular stress block as it applies to flanged sections (Mattock, Kriz and Hognestad 1961). For the current LRFD Specification, the β_1 factor has been removed from the flange overhang term of these equations. See also Seguirant (2002), Girgis, Sun and Tadros (2002), Naaman (2002), Weigel, Seguirant, Brice and Khaleghi (2003), Baran, Schultz and French (2004), and Seguirant, Brice and Khaleghi (2004)

5.7.3.2.3 Rectangular Sections

For rectangular sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used and where the compression flange depth is not less than ~~c~~ $a = b_1 c$ as determined in accordance with Eqs. 5.7.3.1.1.~~3~~4 or 5.7.3.1.2-4, the nominal flexural resistance M_n may be determined by using Eqs. 5.7.3.1.1-1 through 5.7.3.2.2-1, in which case b_w shall be taken as b.

C5.7.3.2.3

5.7.3.2.4 Other Cross-Sections

C5.7.3.2.4

5.7.3.3 LIMITS FOR REINFORCEMENT

5.7.3.3.1 Maximum Reinforcement

Remove all of Article 5.7.3.3.1 and replace with

[PROVISION DELETED IN 2005]

C5.7.3.3

C5.7.3.3.1

Remove all of Article C5.7.3.3.1 and replace with:

In editions of and interims to the LRFD specifications prior to 2005, Article 5.7.3.3.1 limited the tension reinforcement quantity to a maximum amount such that the ratio c/d_e did not exceed 0.42. Sections with $c/d_e > 0.42$ were considered over-reinforced. Over-reinforced nonprestressed members were not allowed, whereas prestressed and partially prestressed members with PPR greater than 50 percent were if "it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved." No guidance was given for what "sufficient ductility" should be, and it was not clear what value of ϕ should be used for such over-reinforced members.

The current provisions of LRFD eliminate this limit and unify the design of prestressed and nonprestressed tension- and compression-controlled members. The background and basis for these provisions are given in Mast (1992). Below a net tensile strain in the extreme tension steel of 0.005, as the tension reinforcement quantity increases, the factored resistance of prestressed and non-prestressed sections is reduced in accordance with Article 5.5.4.2.1. This reduction compensates for decreasing ductility with increasing over-strength. Only the addition of compression reinforcement in conjunction with additional tension reinforcement can result in an increase in the factored flexural resistance of the section.

5.7.3.3.2 Minimum Reinforcement

C5.7.3.3.2

5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

C5.7.3.4

Modify the 3rd paragraph as follows:

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to all bridge decks, ~~transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength~~ and when there is increased concern of appearance and/or corrosion.

5.7.3.5 MOMENT REDISTRIBUTION

Revise Article 5.7.3.5 as follows:

In lieu of more refined analysis, where bonded reinforcement that satisfies the provisions of Article 5.11 is provided at the internal supports of continuous reinforced concrete beams ~~and where the c/d_e ratio does not exceed 0.28~~, negative moments determined by elastic theory at strength limit states may be increased or decreased by not more than ~~the following percentage:~~ 1000 ε_t percent, with a maximum of 20 percent. Redistribution of negative moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

$$\cancel{20\left(1-2.36\frac{c}{d_e}\right)} \quad \cancel{(5.7.3.5\text{-}1)}$$

Positive moments shall be adjusted to account for the changes in negative moments to maintain equilibrium of loads and force effects.

C5.7.3.5

In editions of and interims to the LRFD specifications prior to 2005, Article 5.7.3.5 specified the permissible redistribution percentage in terms of the c/d_e ratio. The current specification specifies the permissible redistribution percentage in terms of net tensile strain ε_t. The background and basis for these provisions are given in Mast (1992).

5.7.3.6 DEFORMATIONS

5.7.3.6 General

5.7.3.6.2 Deflection and Camber

Modify the 1st paragraph and add a new 2nd paragraph as follows:

Instantaneous d~~D~~eflection ~~and camber~~ calculations shall consider appropriate combinations of dead load, live load, prestressing forces, erection loads, concrete creep and shrinkage, and steel relaxation.

Long-term deflection calculations to estimate camber shall consider deflections due to appropriate combinations of all the above mentioned load effects except for those due to live load.

C5.7.3.6

C5.7.3.6

C5.7.3.6.2

Insert the following as paragraph 1

"Camber" is the deflection built into a member, other than by prestressing, in order to achieve a desired grade.

5.7.3.6.2, cont.

Add the following as Commentary to Eq. 1.

Past experiences with cast-in-place box girder bridges show that the design predictions of camber based on I_g are generally in conformance with field measured values.

Replace the current 5th paragraph as follows:

~~Unless a more exact determination is made, the long-time deflection may be taken as the instantaneous deflection multiplied by the following factor:~~

- ~~If the instantaneous deflection is based on~~ ~~I_g: 4.0~~
- ~~If the instantaneous deflection is based on~~ ~~I_e: $3.0-1.2(A'_s/A_s) >= 1.6$~~

Long-term deflection of cast-in-place structures may be calculated by multiplying the instantaneous deflection values based on I_g with the following factors:

- For nonprestressed concrete structures: 4.0
- For prestressed concrete structures: 3.0

Alternatively, long-term deflection of cast-in-place non-prestressed concrete structures may be calculated by multiplying the instantaneous deflection values based on I_e with the following factor:

$$3.0 - 1.2(A'_g/A_s) \geq 1.6 \quad (5.7.3.6.2\text{-}3)$$

where:

A'_s = area of compression reinforcement (in^2)

A_s = area of nonprestressed tension reinforcement (in^2)

C5.7.3.6.3

Revise the existing Commentary as follows:

In prestressed concrete, the long-term deflection ~~is usually~~ may be based on mix-specific data where available, possibly in combination with the calculation procedures in Article 5.4.2.3. Other methods of calculating deflections which consider the different types of loads and the sections to which they are applied, such as that found in (*PCI 1992*), may also be used.

$$3.0 - 1.2(A'_g/A_s) \geq 1.6 \quad (5.7.3.6.2\text{-}3)$$

5.7.3.6.3 Axial Deformation

5.7.4 Compression Members — C5.7.4

5.7.4.1 GENERAL — C5.7.4.1

5.7.4.2 LIMITS FOR REINFORCEMENT — C5.7.4.2

5.7.4.3 APPROXIMATE EVALUATION OF SLENDERNESS EFFECTS — C5.7.4.3

5.7.4.4 FACTORED AXIAL RESISTANCE — C5.7.4.4

5.7.4.5 BIAXIAL FLEXURE — C5.7.4.5

5.7.4.6 SPIRALS AND TIES — C5.7.4.6

5.7.4.7 HOLLOW RECTANGULAR COMPRESSION MEMBERS — C5.7.4.7

5.7.4.7.1 Wall Slenderness Ratio — C5.7.4.7.1

5.7.4.7.2 Limitations on the Use of the Rectangular Stress Block Method — C5.7.4.7.2

5.7.4.7.2a General — C5.7.4.7.2a

5.7.4.7.2b Refined Method for Adjusting Maximum Usable Strain Limit — C5.7.4.7.2b

5.7.4.7.2c Approximate Method for Adjusting Factored Resistance — C5.7.4.7.2c

5.7.5 Bearing	C5.7.5
5.7.6 Tension Members	C5.7.6
5.7.6.1 FACTORED TENSION RESISTANCE	C5.7.6.1
5.7.6.2 RESISTANCE TO COMBINATIONS OF TENSION AND FLEXURE	C5.7.6.2
5.8 SHEAR AND TORSION	C5.8
5.8.1 Design Procedures	C5.8.1
5.8.1.1 FLEXURAL REGIONS	C5.8.1.1
5.8.1.2 REGIONS NEAR DISCONTINUITIES	C5.8.1.2
5.8.1.3 INTERFACE REGIONS	C5.8.1.3
5.8.1.4 SLABS AND FOOTINGS	C5.8.1.4
5.8.2 General Requirements	C5.8.2
5.8.2.1 GENERAL	C5.8.2.1
5.8.2.2 MODIFICATIONS FOR LOW-DENSITY CONCRETE	C5.8.2.2
5.8.2.3 TRANSFER AND DEVELOPMENT LENGTHS	C5.8.2.3
5.8.2.4 REGIONS REQUIRING TRANSVERSE REINFORCEMENT	C5.8.2.4
5.8.2.4 MINIMUM TRANSVERSE REINFORCEMENT	C5.8.2.4
5.8.2.6 TYPES OF TRANSVERSE REINFORCEMENT	C5.8.2.6
5.8.2.7 MAXIMUM SPACING OF TRANSVERSE REINFORCEMENT	C5.8.2.7
5.8.2.8 DESIGN AND DETAILING REQUIREMENTS	C5.8.2.8
5.8.2.9 SHEAR STRESS ON CONCRETE	C5.8.2.9

Modify paragraph 2 as follows:

In determining the web width at a particular level, one-half the diameters of ungrouted ducts or one-quarter the diameter of grouted ducts at that level shall be subtracted from the web width. <u>It is not necessary to reduce b_v for the presence of ducts in fully grouted cast-in-place box girder frames.</u>

5.8.3 Sectional Design Model

C5.8.3

5.8.3.1 GENERAL

C5.8.3.1

5.8.3.2 SECTIONS NEAR SUPPORTS

C5.8.3.2

5.8.3.3 NOMINAL SHEAR RESISTANCE

C5.8.3.3

5.8.3.4 DETERMINATION OF β AND θ

C5.8.3.4

5.8.3.4.1 Simplified Procedure for Nonprestressed Sections

C5.8.3.4.1

5.8.3.4.2 General Procedure

C5.8.3.4.2

5.8.3.4.3 Simplified Procedure

In lieu of Table 5.8.3.4.2-1, β and θ may be evaluated as follows for vertical stirrups:

$$\beta = \frac{4.8}{1 + 1500\varepsilon_x} \quad \text{(5.8.3.4.3-1a US units)}$$

$$\beta = \frac{0.40}{1 + 1500\varepsilon_x} \quad \text{(5.8.3.4.3-1b SI units)}$$

$$\theta = 29 + 7000\varepsilon_x \quad \text{(5.8.3.4.3-2 both US and SI units)}$$

C5.8.3.4.3

Equations 1 and 2 were developed by Micheal Collins and adopted into the Canadian Specs (2004). The effect on resulting values for concrete shear resistance was found to be somewhat greater than the sectional method, but less than those per the 2002 *AASHTO Standard Specifications* on which *Caltrans BDS* is based.

5.8.3.5 LONGITUDINAL REINFORCEMENT

C5.8.3.5

5.8.3.6 SECTIONS SUBJECTED TO COMBINED SHEAR AND TORSION

C5.8.3.6

5.8.3.6.1 Transverse Reinforcement

C5.8.3.6.1

5.8.3.6.2 Torsional Resistance

C5.8.3.6.2

5.8.3.6.3 Longitudinal Reinforcement

C5.8.3.6.3

5.8.4 Interface Shear Transfer – Shear Friction

C5.8.4

5.8.4.1 GENERAL

C5.8.4.1

5.8.4.2 COHESION AND FRICTION

C5.8.4.2

5.8.5 Direct Shear Resistance of Dry Joints

C5.8.5

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.1 General Design Considerations

5.9.1.1 GENERAL

5.9.1.2 SPECIFIED CONCRETE STRENGTHS

5.9.1.3 BUCKLING

5.9.1.4 SECTION PROPERTIES

5.9.1.5 CRACK CONTROL

5.9.1.6 TENDONS WITH ANGLE POINTS OR CURVES

5.9.2 Stresses Due to Imposed Deformation

5.9.3 Stress Limitations for Prestressing Tendons

The following entry in Table 5.9.3-1 shall be changed from:

Prior to Seating	$0.90f_{py}$	$0.90f_{py}$	$0.90f_{py}$

to

Maximum Jacking Stress	$0.90f_{py}$	$\underline{0.75f_{pu}}$ <u>(see note)</u>	$0.90f_{py}$

The following shall be added below Table 5.9.3-1:

<u>Note: For longer frame structures, tensioning to $0.90f_{py}$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value (low relaxation strand, only).</u>

5.9.4 Stress Limits for Concrete

5.9.4.1 FOR TEMPORARY STRESSES BEFORE LOSSES – FULLY PRESTRESSED COMPONENTS

5.9.4.1.1 Compression Stresses

5.9.4.1.2 Tension Stresses

5.9.4.2 FOR STRESSES AT SERVICE LIMIT STATE AFTER LOSSES – FULLY PRESTRESSED COMPONENTS

5.9.4.2.1 Compression Stresses

5.9.4.2.2 Tension Stresses

The following shall replace entries in Table 5.9.4.2.2-1 for "other than segmentally constructed bridges".

C5.9

C5.9.1

C5.9.1.1

C5.9.1.2

C5.9.1.3

C5.9.1.4

C5.9.1.5

C5.9.1.6

C5.9.2

C5.9.3

C5.9.4

C5.9.4.1

C5.9.4.1.1

C5.9.4.1.2

C5.9.4.2.1

C5.9.4.2.2

Table 5.9.4.2.2-1 Tensile Stress Limits in Prestressed concrete at Service Limit State After Losses, Fully Prestressed Components

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only.	No tension
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions, and are located in Caltrans Environmental Areas I or II.	$0.19\sqrt{f'c}$ (KSI) $0.50\sqrt{f'c}$ (MPa)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and are located in Caltrans Environmental Area III.	$0.0948\sqrt{f'c}$ (KSI) $0.25\sqrt{f'c}$ (MPa)
	• For components with unbonded prestressing tendons.	No tension

5.9.5 Loss of Prestress
5.9.5.1 TOTAL LOSS OF PRESTRESS
5.9.5.2 INSTANANEOUS LOSSES
5.9.5.2.1 Anchorage Set
5.9.5.2.2 Friction
5.9.5.2.2a Pretensioned Construction
5.9.5.2.2b Posttensioned Construction
5.9.5.2.3 Elastic Shortening
5.9.5.2.3a Pretensioned Members
5.9.5.2.3b Posttensioned Members
Modify Table 5.9.5.2.3b as follows:

Type of Steel	Type of Duct	K/ft	μ
Wire or strand	Rigid and semi-rigid galvanized metal sheathing Tendon Length:		~~0.15-0.25~~
	≤ 600 FT	0.0002	0.15
	600 FT ≤ 900 FT	0.0002	0.20
	900 FT ≤ 1200 FT	0.0002	0.25
	> 1200 FT	0.0002	>0.25
	Polyethylene	0.0002	0.23
	Rigid steel pipe deviators for external tendons	0.0002	0.25
HS bars	Galvanized metal sheathing	0.0002	0.30

C5.9.5
C5.9.5.1
C5.9.5.1
C5.9.5.2.1
C5.9.5.2.2
C5.9.5.2.2a
C5.9.5.2.2b
C5.9.5.2.3
C5.9.5.2.3a
C5.9.5.2.3
Add as new last paragraph:

For tendon lengths greater than 1200 feet, investigation is warranted on current field data of similar length frame bridges for appropriate values of μ.

5.9.5.3 APPROXIMATE ESTIMATE OF TIME-DEPENDENT LOSSES

Add the following to the end of Article 5.9.5.3:

For ~~cast-in-place~~ post-tensioned members, the approximate estimate of time-dependent losses may be taken as the lump sum value of 25 ksi.

C5.9.5.3

Add the following to the end of Article C5.9.5.3:

The expressions for estimating time-dependent losses in Table 5.9.5.3-1 were developed for pretentioned ~~precast~~ members and should not be used for ~~cast-in-place~~ post-tensioned structures. Preliminary research at UCSD indicates that the time-dependent losses for cast-in-place post-tensioned structures are between 25 ksi and 30 ksi. Until the research is completed, and, in lieu of a more detailed analysis, a lump sum value for losses in post-tensioned members is provided.

Table 5.9.5.3-1 Time-Dependent Losses in ksi.

Type of Beam Section	Level	For Wires and Strands with f_{pu} = 235, 250 or 270 ksi	For Bars with f_{pu} = 145 or 160 ksi
Rectangular Beams, Solid Slab	Upper Bound Average	29.0 + 4.0 *PPR* 26.0 + 4.0 *PPR*	19.0 + 6.0 *PPR*
Pretensioned Box Girder	Upper Bound Average	21.0 + 4.0 *PPR* 19.0 + 4.0 *PPR*	15.0
Single T, Double T, Hollow Core and Voided Slab	Upper Bound Average	$39.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$ $33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$	$31.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\,PPR$

5.9.5.4 REFINED ESTIMATES OF TIME-DEPENDENT LOSSES

C5.9.5.

5.9.5.4.1 General

C5.9.5.4.1

5.9.5.4.2 Losses: Time of Transfer to Time of Deck Placement

5.9.5.4.3 Losses: Time of Deck Placement to Final Time

5.9.5.4.4 Precast Pretensioned Girders without Composite Topping

C5.9.5.4.4

5.9.5.4.5 Post-tensioned Non-segmental Girders

C5.9.5.4.5

5.9.5.5 LOSSES FOR DEFLECTION CALCULATIONS

C5.9.5.5

5.10.9.3.2 Design Principles
5.10.9.3.3 Special Anchorage Devices
5.10.9.3.4 Intermediate Anchorages
5.10.9.3.4a General
5.10.9.3.4b Tie Backs
5.10.9.3.4c Blister and Rib Reinforcement
5.10.9.3.5 Diaphragms
5.10.9.3.6 Multiple Slab Anchorages
5.10.9.3.7 Deviation Saddles
5.10.9.4 APPLICATION OF THE STRUT-AND-TIE MODEL TO THE DESIGN OF GENERAL ZONE
5.10.9.4.1 General
5.10.9.4.2 Nodes
5.10.9.4.3 Struts
5.10.9.4.4 Ties
5.10.9.5 ELASTIC STRESS ANALYSIS
5.10.9.6 APPROXIMATE STRESS ANALYSES AND DESIGN
5.10.9.6.1 Limitations of Application
5.10.9.6.2 Compressive Stresses
5.10.9.6.3 Bursting Forces
5.10.9.6.4 Edge Tension Forces
5.10.9.7 DESIGN OF LOCAL ZONES
5.10.9.7.1 Dimensions of Local Zone
5.10.9.7.2 Bearing Resistance
5.10.9.7.3 Special Anchorage Devices
5.10.10 Pretensioned Anchorage Zones
5.10.10.1 FACTORED BURSTING RESISTANCE
5.10.10.2 CONFINEMENT REINFORCEMENT
5.10.11 Provisions for Seismic Design
5.10.11.1 GENERAL
5.10.11.2 SEISMIC ZONE 1
5.10.11.3 SEISMIC ZONE 2
5.10.11.4 SEISMIC ZONES 3 AND 4
5.10.11.4.1 Column Requirements
5.10.11.4.1a Longitudinal Reinforcement
5.10.11.4.1b Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4. The column shall be investigated for both extreme load cases, as specified in Article 3.10.8, at the extreme event limit state. ~~The resistance factors of Article 5.5.4.2 shall be replaced for both spirally and tied reinforced axial compression members by the value of 0.50. For compression members with flexure, the value of ϕ may be increased linearly from 0.50 to 0.90 as the factored axial resistance, ϕP_n, decreased from $0.20 f'_c A_g$ and 0.0.~~

C5.10.9.3.2
C5.10.9.3.3
C5.10.9.3.4
C5.10.9.3.4a
C5.10.9.3.4b
C5.10.9.3.4c
C5.10.9.3.5
C5.10.9.3.6
C5.10.9.3.7
C5.10.9.4
C5.10.9.4.1
C5.10.9.4.2
C5.10.9.4.3
C5.10.9.4.4
C5.10.9.5
C5.10.9.6
C5.10.9.6.1
C5.10.9.6.2
C5.10.9.6.3
C5.10.9.6.4
C5.10.9.7
C5.10.9.7.1
C5.10.9.7.2
C5.10.9.7.3
C5.10.10
C5.10.10.1
C5.10.10.2
C5.10.11
C5.10.11.1
C5.10.11.2
C5.10.11.3
C5.10.11.4
C5.10.11.4.1
C5.10.11.4.1a
C5.10.11.4.1b

Figure 1 has been deleted.

5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT	C5.11
5.11.1 General	C5.11.1
5.11.1.1 BASIC REQUIREMENTS	C5.11.1.1
5.11.1.2 FLEXURAL REINFORCEMENT	C5.11.1.2
5.11.1.2.1 General	C5.11.1.2.1
5.11.1.2.2 Positive Moment Reinforcement	C5.11.1.2.2
5.11.1.2.3 Negative Moment Reinforcement	C5.11.1.2.3
5.11.1.2.4 Moment Resisting Joints	C5.11.1.2.4
5.11.2 Development of Reinforcement	C5.11.2
5.11.2.1 DEFORMED BARS AND DEFORMED WIRE IN TENSION	C5.11.2.1
5.11.2.1.1 Tension Development Length	C5.11.2.1.1
5.11.2.1.2 Modification Factors That Increase	C5.11.2.1.2
5.11.2.1.3 Modification Factors That Decrease	C5.11.2.1.3
5.11.2.2 DEFORMED BARS IN COMPRESSION	C5.11.2.2
5.11.2.2.1 Compressive Development Length	C5.11.2.2.1
5.11.2.2.2 Modification Factors	C5.11.2.2.2
5.11.2.3 BUNDLED BARS	C5.11.2.3
5.11.2.4 STANDARD HOOKS IN TENSION	C5.11.2.4
5.11.2.4.1 Basic Hook Development Length	C5.11.2.4.1
5.11.2.4.2 Modification Factors	C5.11.2.4.
5.11.2.4.3 Hooked-Bar Tie Requirements	C5.11.2.4.3
5.11.2.5 WELDED WIRE FABRIC	C5.11.2.5
5.11.2.5.1 Deformed Wire Fabric	C5.11.2.5.1
5.11.2.5.2 Plain Wire Fabric	C5.11.2.5.2
5.11.2.6 SHEAR REINFORCEMENT	C5.11.2.6
5.11.2.6.1 General	C5.11.2.6.1
5.11.2.6.2 Anchorage of Deformed Reinforcement	C5.11.2.6.2
5.11.2.6.3 Anchorage of Wire Fabric Reinforcement	C5.11.2.6.3
5.11.2.6.4 Closed Stirrups	C5.11.2.6.4
5.11.3 Development by Mechanical Anchorages	C5.11.3
5.11.4 Development of Prestressing Strand	C5.11.4
5.11.4.1 GENERAL	C5.11.4.1
5.11.4.2 BONDED STRAND	C5.11.4.2

5.11.4.3 PARTIALLY DEBONDED STRANDS

C5.11.4.3

Revise the 2nd, 3rd, and 4th paragraphs as follows:

The number of partially debonded strands should not exceed ~~25~~ 33 percent of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed ~~40~~ 50 percent of the strands in that row.

The length of debonding of any strand shall be such that all limit states are satisfied with consideration of the total developed resistance at any section being investigated. Not more than 40 percent of the debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any section.

Insert the following as paragraph 3 in Article 5.13.4.5.2 (on AASHTO '05 ballot):

For cast-in-place concrete piling, clear distance between parallel longitudinal and parallel transverse reinforcing bars shall not be less than 5 times the maximum aggregate size or 5 inches, except as noted in Article 5.13.4.6 for seismic requirements.

5.13.4.6 SEISMIC REQUIREMENTS

5.13.4.6.1 Zone 1

5.13.4.6.2 Zone 2

5.13.4.6.2a General

5.13.4.6.2b Cast-in-Place Piles

Revise Article 5.13.4.6.2b as follows (on '05 AASHTO ballot):

"For cast-in-place piles, longitudinal steel shall be provided….For piles less than 24-IN in diameter, ~~S~~spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0 IN, except that the pitch shall not exceed ~~3~~ 4.0 IN within a length below the pile cap reinforcement of not less than 2.0 FT or 1.5 pile diameters , whichever is greater ~~below the pile cap reinforcement~~. See Article 5.10.11.3.

5.13.4.6.2c Precast Reinforced Piles

5.13.4.6.2d Precast Prestressed Piles

5.13.4.6.3 Zones 3 and 4

5.13.4.6.3a General

C5.13.4.6

C5.13.4.6.1

C5.13.4.6.2

C5.13.4.6.2a

C5.13.4.6.2b

Add the following as C5.13.4.6.2b

Cast-in-place concrete pilings may only have been vibrated directly beneath the pile cap, or in the uppermost sections. Where concrete is not vibrated, nondestructive tests in the State of California have shown that voids and rock pockets form when adhering to maximum confinement steel spacing limitations from some seismic recommendations. Concrete does not readily flow through the resulting clear distances between bar reinforcing, weakening the concrete section, and compromising the bending resistance to lateral seismic loads. Instead of reduced bar spacing, bar diameters should be increased which results in larger openings between the parallel longitudinal and transverse reinforcing steel.

C5.13.4.6.2

C5.13.4.6.2d

C5.13.4.6.3

C5.13.4.6.3a

5.13.4.6.3b Confinement Length

C5.13.4.6.3b

5.13.4.6.3c Volumetric Ratio for Confinement

C5.13.4.6.3c

5.13.4.6.3d Cast-in-Place Piles

Revise Article 5.13.4.6.3d as follows(on '05 AASHTO ballot):

"For cast-in-place piles, longitudinal steel shall be provided….For piles less than 24-IN in diameter, ~~S~~spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0-IN ~~pitch~~, except that the pitch shall not exceed 4.0 IN ~~for the top~~ within a length below the pile cap reinforcement of not less than 4.0 FT or two pile diameters, whichever is greater, ~~where the pitch shall be 3~~ ~~4.0 IN~~ and where the volumetric ratio and splice details shall conform to Articles 5.10.11.4.1d and e."

C5.13.4.6.3d

Add the following as C5.13.4.6.3d

Cast-in-place concrete pilings may only have been vibrated directly beneath the pile cap, or in the uppermost sections. Where concrete is not vibrated, nondestructive tests in the State of California have shown that voids and rock pockets form when adhering to maximum confinement steel spacing limitations from some seismic recommendations. Concrete does not readily flow through the resulting clear distances between bar reinforcing, weakening the concrete section, and compromising the bending resistance to lateral seismic loads. Instead of reduced bar spacing, bar diameters should be increased which results in larger openings between the parallel longitudinal and transverse reinforcing steel.

5.14 PROVISIONS FOR STRUCTURE TYPES

C5.14

5.14.1 Beams and Girders

C5.14.1

5.14.1.2 Precast Beams

C5.14.1.2

5.14.1.2.1 Preservice Conditions

C5.14.1.2.1